ERDC/CHL TR-01-3

Coastal and Hydraulics Laboratory

US Army Corps of Engineers®
Engineer Research and Development Center

Kaumalapau Harbor, Lanai, Hawaii, Two-Dimensional Breakwater Stability Study

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Preface

The model investigation of Kaumalapau Harbor, HI, reported herein, was requested by the U. S. Army Engineer Division, Pacific Ocean, and conducted at the Coastal and Hydraulic Laboratory (CHL) of the U.S. Army Engineer Research Development Center (ERDC). Authorization for ERDC to perform the study was granted by the Pacific Ocean Division.

Model experiments were conducted at ERDC during the period September 1999 through November 1999 by personnel of the Harbors and Entrances Branch (HNH), CHL, under the supervision of Dr. James R. Houston and Mr. Thomas Richardson, former Director and Assistant Director of CHL, respectively; and the direct supervision of Mr. Dennis Markle, Chief of HNH. Experiments were conducted by Messrs. Willie Dubose, Civil Engineering Technician; Raymond Reed, Civil Engineering Technician; David Daily, Instrumentation Services Technician; and Ernest R. Smith, Research Hydraulic Engineer. Mr. Smith prepared this report.

Liaison was maintained with the Pacific Ocean Division through monthly progress reports and telephone conversations during the course of the investigation. Mr. Scott Sullivan, Sea Engineering, visited ERDC during experiments of Plans 2 through 5 to observe the model and give guidance on stability plans to study. During Plans 4 and 5, Mr. Stan Boc and Ms. Helen Stupplebeen, Pacific Ocean Division, and Mr. Fred Nunes, Harbors Division, Hawaii Department of Transportation, visited ERDC to observe model experiments and discuss model results.

At the time of publication of this report, Dr. James R. Houston was Director of ERDC, and COL James S. Weller, EN, was Commander.

1 Introduction

Background

At the request of the U.S. Army Engineer Division, Pacific Ocean, the U.S. Army Engineer Research and Development Center's (ERDC) Coastal and Hydraulics Laboratory (CHL) carried out a two-dimensional (2-D) breakwater stability study for Kaumalapau Harbor, Lanai, HI. Kaumalapau Harbor, Lanai's main commercial harbor, is being studied for development. It is protected from the open ocean by a single rubble-mound breakwater approximately 76.2 m (250 ft) long.

The breakwater, particularly the seaward end, has been damaged by previous storms, and, at present, the most seaward 22.9 m (75 ft) of breakwater length has been reduced to a submerged mound. The existing breakwater does not provide adequate protection of the harbor from approaching ocean waves, primarily in the vicinity of the main dock. Difficult wave conditions are reported to occur during the winter season, particularly during high-energy swell from the north.

Study Location

Kaumalapau Harbor is located in a small embayment along the south central part of the west coast of the island of Lanai (Figure 1). The northwest lobe of Lanai shelters the site from the north and north-northwest, and general exposure is to the west. The Kaumalapau Harbor entrance is formed between a rocky point on the south side and the main Kaumalapau breakwater on the north side (Figure 2). Navigation lights are located on the breakwater tip and the point south of the harbor.

The harbor entrance is 183 m (600 ft) wide, opening into a semiprotected, 40,500-m^2 (10-acre) berthing area. Water depths in the berthing area range from 9.1 to 15.2 m (30 to 50 ft). The rubble-mound breakwater is approximately 76.2 m (250 ft) long with a crest elevation of about 3.0 m (10 ft). Commercial operations occur along a single 121.9-m (400-ft) wharf in the lee of the breakwater.

Kaumalapau Harbor is Lanai's main commercial harbor. Lanai's second harbor of note is a small boat harbor at Manele Bay on the southeast coast.

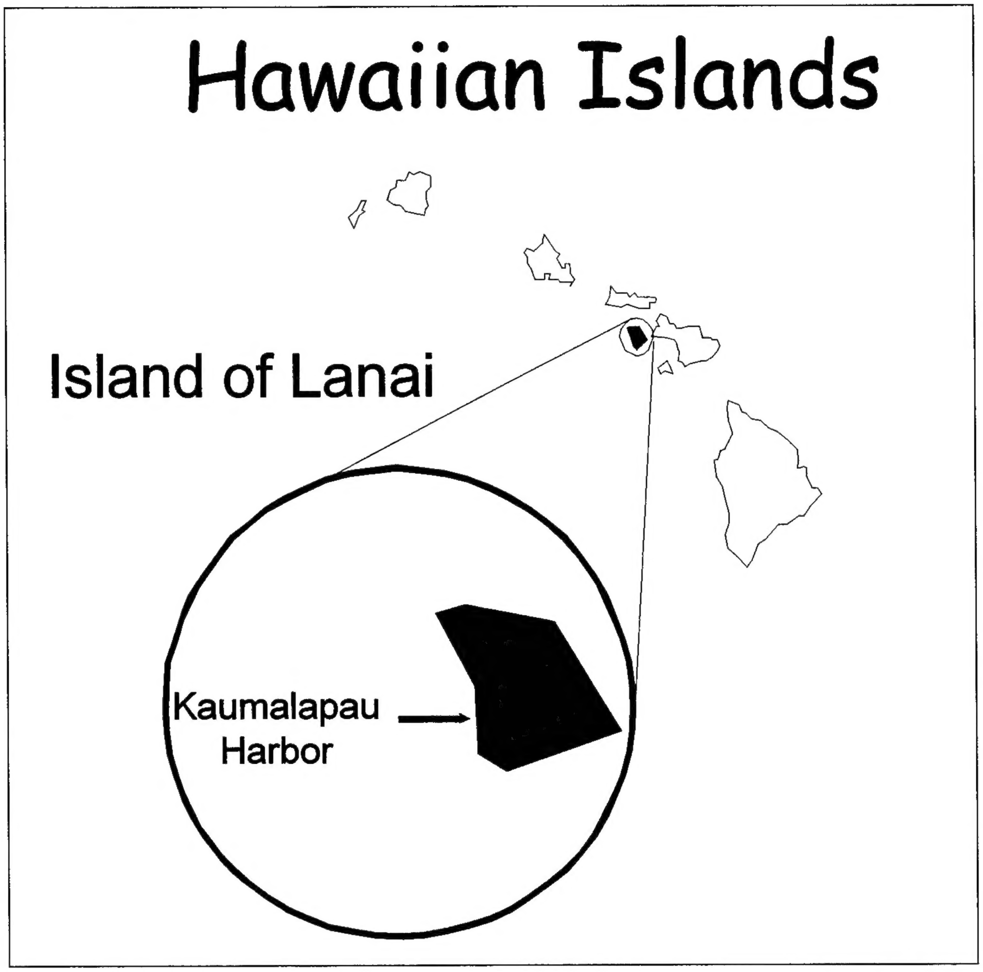

Figure 1. Study location

Primary cargos at Kaumalapau have traditionally been outbound pineapples and inbound supplies needed for people and operations involved in pineapple farming. Pineapple farming on Lanai has virtually ended, and facilities for recreation and tourism are developing. Kaumalapau Harbor will continue to be the critical sea link between Lanai's residents and visitors and the rest of the world.

Results of Previous Stability Study

A previous study of Kaumalapau Harbor was conducted to determine a stable breakwater configuration to reduce wave heights along the main dock area without adversely impacting harbor navigation (Smith 1998). The study consisted of four interrelated aspects which include the following:

a. Review of wave hindcast data.

b. Field wave gauging.

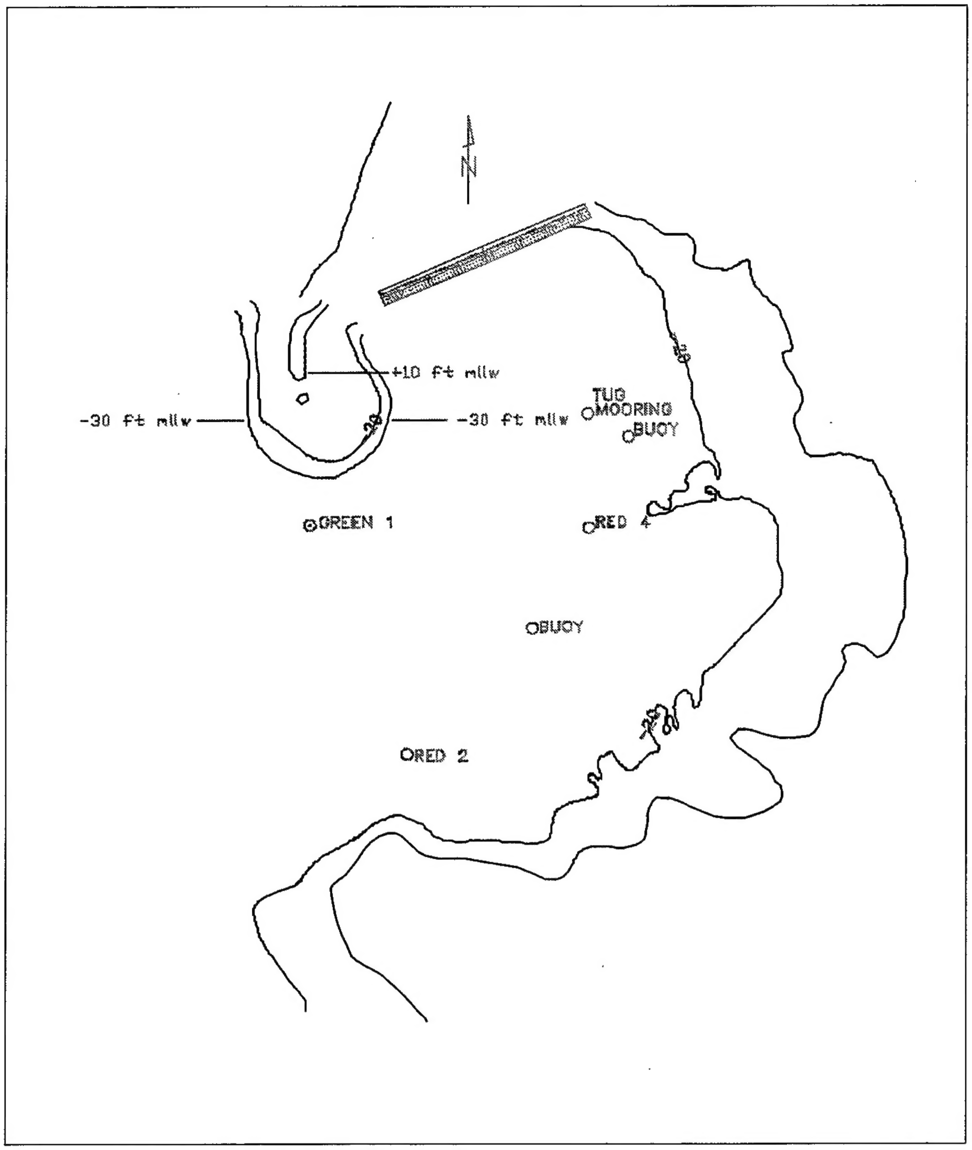

Figure 2. Kaumalapau Harbor

c. Numerical model simulations.

d. Physical model studies.

Three-dimensional (3-D) physical model studies to evaluate harbor response to short-period waves and to evaluate stability of armor units placed on the breakwater were conducted in the same basin having a prototype depth of 29 m (95 ft) mean lower low water (mllw) at a 1:49 undistorted linear scale. The design wave height for the stability study was specified in a reconnaissance report as a 9.8-sec, 8.5-m (28-ft) deepwater wave and 7.1 m (23.4 ft) at the structure (U.S. Army Engineer District, Honolulu, 1993). The proposed breakwater consisted of CORE-LOC ® (hereafter referred to as Core-Loc) armor units placed on the existing structure, and the units extended from -10.7 m

(-35.1 ft) mllw on the sea side and –6.4 m (-21.0 ft) mllw on the lee side up to the crest elevation of +6.1 m (+20.0 ft) mllw. The crown width was 6.1 m (20 ft) wide and consisted of a rib cap. A sketch of typical cross section examined in the previous study is shown in Figure 3. Stability experiments were conducted on the proposed breakwater for waves exceeding the design height for 9.8- and 12-sec periods from 221- and 251-deg wave directions (Table 1). Core-Loc armor units were used in the stability study and placement density of units was defined by the following equation:

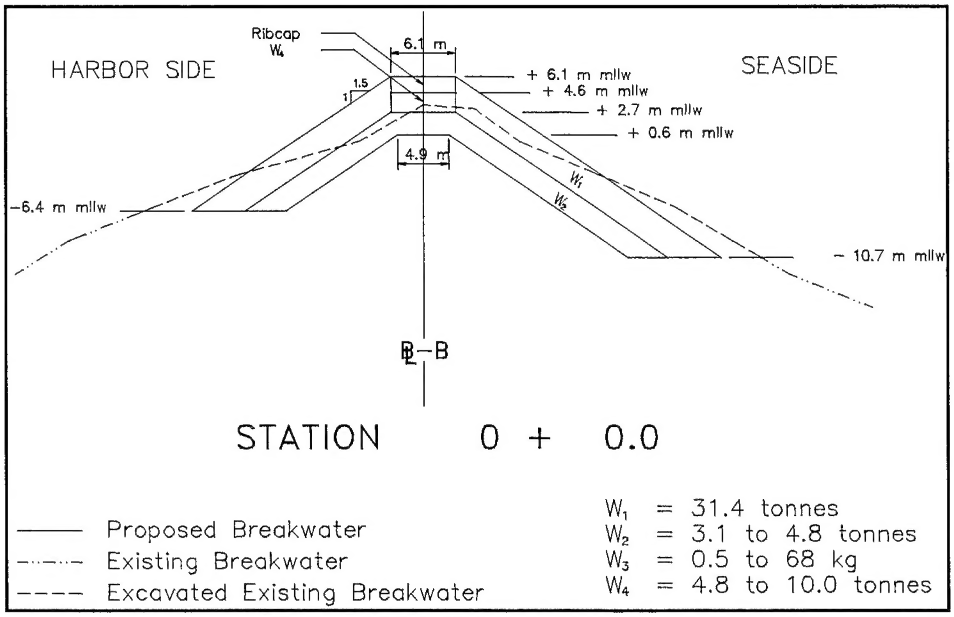

Figure 3. Typical cross section of proposed breakwater in previous study (Smith 1998)

Table 1
Wave Conditions Used in Previous Study (from Smith 1998)

T_p Sec	H_o m	H' m	H_d (221 deg) m	H_d (251 deg) m	Duration sec (model)
9.8	3.6	3.0	3.2	3.0	880
9.8	5.5	4.6	4.8	4.4	880
9.8	7.3	6.1	6.4	5.9	880
9.8	8.5[1]	7.2	7.5[1]	6.9[1]	880
9.8	9.1	7.6	8.3	7.3	880
9.8	9.9	8.3	8.7	8.0	880
12.0	5.5	4.6	4.3	4.3	880
12.0	7.3	6.1	5.8	5.7	880
12.0	8.5	7.2	6.8	6.7	880
12.0	9.1	7.6	7.3	7.2	880
12.0	10.8	9.1	8.6	8.5	880
16.0	4.9	4.6	3.9	4.5	880
16.0	6.6	6.1	5.2	6.0	880
16.0	8.5	7.9	6.7	7.7	880

[1] Design wave height determined by Pacific Ocean Division.
H_o = deepwater zero-moment wave height
H' = zero-moment wave height at wave generator depth (28.9-m (95-ft) prototype)
H_d = average zero-moment wave height at structure

$$\frac{N}{A} = \phi V^{\frac{2}{3}} \tag{1}$$

where N is the number of units in a given area, A, ϕ is the packing density coefficient, and V is the armor unit volume. Results of basic research tests indicated that the optimum packing density coefficient for Core-Locs is $0.58 < \phi < 0.62$.

Nine plans were evaluated for stability in the previous study. It was found that densely placed (ϕ =0.63) 18.1-tonne (20-ton) Core-Locs placed on the sea side were stable for design wave conditions, but the section was damaged for wave heights exceeding the design condition. A heavier Core-Loc, 31.4 tonnes (34.6 tons) placed on the head and harbor side placed at ϕ =0.62, was stable for wave heights exceeding the design condition.

Since completion of the previous study, a Value Engineering meeting between U.S. Army Corps of Engineers personnel and private consultants was held to improve the project. Of the options considered, the following recommendations were considered for review:

a. Modification of rubble mound.

b. Replace rubble mound with caissons.

c. Re-evaluate concrete rib design.

The group concluded that there was a risk in replacing the rubble-mound structure with caissons because the caissons would be placed on the existing foundation, which is somewhat unknown. Additionally, the cost savings were approximately the same as estimated savings for the modified rubble-mound structure. Because of the minimal cost savings and added risk, using caissons was not considered as an alternative.

The proposed change to the rib cap design consisted of decreasing the concrete ties from 0.9 m wide by 0.9 m deep (3 ft wide by 3 ft deep) to 0.9 m wide by 0.46 m deep (3 ft wide by 1.5 ft deep). The proposed change followed the precedent of previous rib cap designs and should provide adequate strength.

Modifications to the rubble mound consisted of raising the toe elevation on the trunk from -12.2 m (-40 ft) mllw to –9.1 m (-30 ft) mllw, decreasing the crest elevation from +6.1 m (+18 ft) mllw to +3.7 m (+12 ft) mllw, increasing Core-Loc size from 25.5 tonnes (28 tons) to 30.9 tonnes (34 tons), lowering the recommended Core-Loc packing density coefficient from 0.62 to 0.58, and decreasing the median underlayer size from 20 percent of the armor layer weight to 10 percent. Modifications to the rubble mound would require a physical model study to verify toe and crest stability on the trunk and head.

In addition to the proposed changes, the design wave height was discussed. A thorough analysis of the design wave had not been conducted for survivability

prior the study reported in Smith 1998. The previous design wave was based on a diminishing hurricane wind speed and pressure. The wave was assumed to break in the design depth of the structure of 9.1 m (30 ft) mllw and the depth-limited design wave at the structure was determined using solitary wave theory (breaker height to breaker depth ratio equal to 0.78). No risk analysis was conducted so it was not known if a diminishing hurricane was the most likely or worst case hurricane and the diminishing hurricane may not be conservative. Additionally, the depth of the structure is 18.3 m (60 ft) mllw and the offshore bathymetry slopes seaward at 1V:10H, indicating the breaker depth ratio should be much higher than 0.78. The fronting slope and deeper depth would result in a much higher wave than was originally provided for the previous study.

Purpose of Present Study

The purpose of the study was to examine the effects of design level and above wave conditions on the stability of the proposed breakwater rehabilitation. Because of time and budget limitations, a 2-D physical model of the proposed Kaumalapau breakwater was constructed and contained a 1V:10H fronting slope to represent the seaward bathymetry of the prototype. Results of the 2-D study were compared to results of the previous 3-D study and inferences were made to estimate the effects the modified wave and bathymetrical conditions would have on the head.

This report describes the revised breakwater design, facilities used, and results of the stability experiments. A description of the model and experiment procedures is found in Chapter 2. Results of the study are given in Chapter 3, and conclusions are listed in Chapter 4. Appendix A includes symbol notation used in the report.

2 Physical Model

Model Design

Two-dimensional stability experiments were conducted at a geometrically undistorted linear scale of 1:54.3, model to prototype, for the proposed breakwater cross section at Kaumalapau Harbor, HI. Scale was based on size availability of model armor units and the capabilities of the available wave generator to produce required wave heights for the selected water depth.

Because the specific weights of water and armor layer material differed between the model and prototype, the transference equation of Hudson (1975) was used to determine the model scale that most closely represented desired prototype weights for the Core-Locs available at ERDC:

$$\frac{(W_a)_m}{(W_a)_p} = \frac{(\gamma_a)_m}{(\gamma_a)_p} \left(\frac{l_m}{l_p} \right)^3 \left[\frac{(S_a)_p - 1}{(S_a)_m - 1} \right]^3 \tag{2}$$

in which the subscripts m and p refer to model and prototype quantities, respectively, W_a is the weight of individual armor or stone, γ_a is the specific weight of an individual armor unit or stone, l_m/l_p is the linear scale of the model, S_a is the specific gravity of an individual armor unit or stone relative to the water in which it is placed, $S_a = \gamma_a/\gamma_w$, and γ_w is the specific weight of water.

A 1:54.3 scale, $(\gamma_a)_p = 2320$ kg/m^3 (145 lb/ft^3), and model Core-Loc weights of 104- and 186-g used in Equation 2 yielded prototype weights of 17.8 and 32.2 tonnes (19.6 and 35.5 tons), respectively, and were used as armor layer, W_1. Time relations were scaled according to Froude Model Law (Stevens et al. 1942), and model to prototype relations were derived in terms of l and t shown in Table 2.

Table 2
Model-Prototype Scale Relations (1:54.3 scale)

Characteristic	Dimension	Scale Relations Model:Prototype
Length	l	l_r = 1:54.3
Area	l^2	a_r = 1:2948.5
Volume	l^3	v_r = 1:160103

Scale effects of viscous forces associated with flow through the underlayer and core of the proposed breakwater were addressed using the method of Keulegan (1973) to assure that flow through the model structure was turbulent. The proposed underlayer size of the Kaumalapau breakwater is to be one-tenth of the armor size, and the structure is to be constructed on the existing breakwater using the existing material as the core. Stone weights of 9.6 to 15.2 g (0.021 to 0.034 lbs),12.0 to 22.6 g (0.026 to 0.050 lbs) and 22.6 to 44.4 g (0.050 to 0.098 lbs) represented prototype sizes of 2.2 to 3.4 tonnes (2.4 to 3.8 tons), 2.7 to 5.1 tonnes (3.0 to 5.5 tons), and 5.1 to 10.0 tonnes (5.6 to 11.0 tons) for underlayer, W_2, toe stone, W_3, and core (existing breakwater) material, W_4, and armor capstone, W_5, respectively. Model underlayer, core material, and armor capstone, remained the same for all stability plans. Model materials for each layer, and the associated prototype size, are shown in Table 3.

Table 3
Prototype and Model Material Sizes and Thicknesses

	Model		Prototype	
Layer	Width cm (ft)	Weight g (lbs)	Width m (ft)	Weight tonnes (tons)
W_a (Plan 1)	5.5 (0.18)	104 (0.23)	3.0 (9.9)	17.8 (19.6)
W_a (Plans 2 through 5)	6.6 (0.22)	186 (0.41)	3.6 (11.9)	32.2 (35.5)
W_2	3.9 (0.13)	9.6 to 15.2 (0.021 to 0.034)	2.1 (6.9)	2.2 to 3.4 (2.4 to 3.8)
W_3	4.8 (0.16)	12.0 to 22.6 (0.026 to 0.050)	2.6 (8.5)	2.7 to 5.1 (3.0 to 5.5)
W_4	-	22.6 to 44.4 (0.050 to 0.098)	-	5.1 to 10.0 (5.6 to 11.0)
W_5	1.3 (0.04)	22.6 to 44.4 (0.050 to 0.098)	0.7 (2.3)	5.1 to 10.0 (5.6 to 11.0)

Experiment Facilities and Equipment

Experiments were performed in a 61.1-m-long, 1.52-m-wide, 2.0-m-deep (200-ft-long, 5-ft-wide, 6-ft-deep) wave tank. Figure 4 shows tank dimensions, bottom slopes, wave gauge placement, and structure location for the study. The tank contained a concrete-capped compound slope to represent idealized local bathymetry seaward of the breakwater location. A 19.5-m-long (5.9-ft-long), 1V:44H slope originated at a model distance of 2.4 m (7.9 ft) from the wave board and represented a prototype depth of 77.0 m (252.6 ft) mllw. The 1V:44H slope terminated at a prototype depth of 52.9 m (173.6 ft) mllw, and was horizontal for 12.2 m (40.0 ft). A 1V:10H, which represented the nearshore bathymetry of the structure, extended for 6.1 m (20 ft) to a prototype depth of 19.8 m (65 ft) mllw. The model breakwater was constructed at the terminus of the 1V:10H slope on a 6.1-m-long (20-ft-long) horizontal section. Plywood was placed at the toe of the 1V:10H slope and extended to the end of the horizontal section to separate the tank into a 0.9-m-wide (3-ft-wide) section near the glass and a 0.6-m-wide (2-ft-wide) section near the far tank wall. The purpose of the plywood wall was to reduce the effect of re-reflected waves from the breakwater.

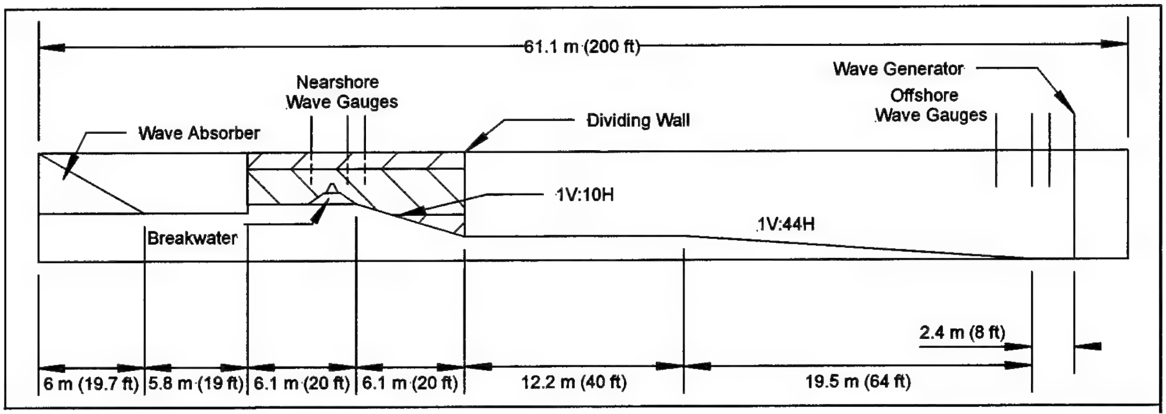

Figure 4. Wave tank used for study

Waves were generated by a piston-type electronically controlled electro-hydraulic system. Displacement of the wave board was controlled by an irregular wave command signal transmitted to the wave board by a microcomputer. Waves were produced by the periodic displacement of the wave board.

Water surface elevations were recorded using six single wire capacitance-type gauges, sampled at 20 Hz. The gauges were arranged in two groups of three gauges to obtain incident and reflected wave heights by the method of Goda and Suzuki (1976). The first array was positioned near the wave board to obtain offshore wave heights. The second array was positioned at the breakwater toe location during wave calibration, and was positioned on the other side of the dividing wall but in the same onshore location of the structure during stability experiments.

Prior to conducting stability experiments, the facility was calibrated without the breakwater in place for selected design wave periods at still-water levels of +1.5 m and 0.67 m (+5 ft and +2.2 ft) mllw, respectively. Breakwater stability experiments were conducted for the wave conditions listed in Tables 4 and 5. Table 4 shows wave conditions used for Plan 1, and Table 5 lists wave conditions for Plans 2 through 5; however, not all of the conditions shown in Table 5 were used for every plan. The waves used for each plan are designated by an X in Table 5. All waves heights listed in Tables 4 and 5 refer to zero-moment wave height, $H_{mo,}$ which in deep water is very similar to the significant wave height, H_s, defined as the average of the highest one-third of the waves in a wave train.

Model Construction

The proposed breakwater is to be constructed on top of the existing structure and is to consist of an armor layer, an underlayer, and a core using existing material of the original structure.

Construction of the model breakwater simulated prototype construction as closely as possible. The core and underlayer material were dumped by shovel,

Table 4 Original Design Wave Conditions (swl = +1.5 m (+5 ft) mllw)			
T_p sec	H_o m (ft)	H_d m (ft)	Duration sec (model)
9.8	3.6 (11.8)	3.0 (9.8)	900
9.8	5.5 (18.0)	4.4 (14.4)	900
12.0	5.5 (18.0)	4.3 (14.1)	900
16.0	4.9 (16.0)	4.5 (14.8)	900
9.8	7.3 (24.0)	5.9 (19.4)	900
12.0	7.3 (24.0)	5.7 (18.7)	900
16	6.6 (21.7)	6.0 (19.7)	900
9.8	8.5 (28.0)	6.9 (22.6)	900
12.0	8.5 (28.0)	6.7 (22.0)	900
16.0	8.5 (28.0)	7.7 (25.3)	900
9.8	9.1 (30.0)	7.3 (24.0)	900
12.0	9.1 (30.0)	7.2 (23.6)	900
9.8	9.9 (32.5)	8.0 (26.2)	900
12.0	10.8 (35.4)	8.5 (28.0)	900
Note: Design wave height determined by Pacific Ocean Division. H_o - deepwater zero-moment wave height H_d - average zero-moment wave height at structure			

Table 5 Modified Design Wave Conditions (swl = +0.67 m (+2.2 ft) mllw)						
$T_p$sec	H_d m (ft)	Plan 2	Plan 3	Plan 4	Plan 5	Duration sec (model)
20.0	1.8 (6.0)	X	X	X	X	900
9.8	3.0 (10.0)	X		X	X	900
12.0	3.0 (10.0)	X	X	X	X	900
9.8	4.6 (15.0)	X	X	X	X	900
12.0	4.6 (15.0)	X	X	X	X	900
12.0	6.1 (20.0)	X	X	X	X	900
16.0	6.1 (20.0)	X	X			900
12.0	7.6 (25.0)	X	X	X	X	900
16.0	7.6 (25.0)			X	X	900
12.0	9.1 (30.0)	X	X	X	X	900
16.0	9.1 (30.0)	X	X	X	X	900
12.0	10.7 (35.0)	X	X	X	X	900
16.0	10.7 (35.0)			X	X	900
12.0	12.2 (40.0)	X	X	X	X	900
16.0	12.2 (40.0)			X	X	900
H_d - average zero-moment wave height at structure						

smoothed to grade, and compacted with hand trowels to simulate consolidation that would have occurred due to wave action.

The armor layer was comprised of either 17.8- or 32.2-tonne (19.6- or 35.5-ton) Core-Locs placed on a 1V:1.5H slope. Core-Loc armor units were placed in a single armor layer using a selective random placement described by Melby and Turk (1995). The first row of Core-Locs were aligned with vertical

flukes and abutting the adjacent units. The second row of units was placed in a manner that the flukes overlapped the waist portion of the first row units. Units above the first two rows were placed in a random fashion with the exception that no unit was placed on the slope with vertical flukes directly above a unit also placed with vertical flukes. Basic 2-D research tests with Core-Locs have shown that two units placed with flukes oriented vertically and one placed atop the other do not interlock and can form a weak spot in the armor layer.

The number of Core-Locs placed on the breakwater was based on Equation 1. Plan 1, the proposed breakwater from Smith (1998), was constructed using a packing density value, ϕ, of 0.62. During construction of Plans 2 through 5, ϕ varied between 0.58 and 0.59.

A rib cap was placed on the crown of the structure, and it was assumed that the rib cap would be stable in the prototype. Therefore, it was not necessary that the cap be dynamically similar to the prototype. The model rib cap, constructed of Plexiglas, was geometrically similar to the prototype and was anchored to the sidewalls in the model to ensure proper transmission, reflection, and dissipation of wave energy. Individual ribs were 0.9-m (3-ft) wide, 0.5-m (1.5-ft) high, 6.1-m (20-ft) long and spaced 1.8 m (6 ft) on centers. The ribs were oriented at a 90-deg angle to the longitudinal axis of the breakwater. The rib cap included 0.6-m-wide runners placed 1.5 m on center from the rib ends. A sketch showing rib cap dimensions is shown in Figure 5.

Experiment Procedures

Photographs were taken of the side view, harbor side, and sea side before testing was initiated without water in the tank. Following before-test photographs, the tank was flooded to the appropriate water depth and the structure was exposed to 9.8-sec, 3.0-m (10-ft) waves. These lower waves provided overtopping information and allowed settling and nesting of the newly constructed section, which would occur under typical daily wave conditions prior to being exposed to a design level storm. The remaining storm conditions were reproduced upon completion of the lower waves. Response of the structure was recorded during and after each wave condition. Effects of waves on individual units, toe stability, and the general condition of the breakwater were recorded. The tank was drained upon completion of the entire storm series and after test photographs were taken of the side view, harbor side, and sea side.

Visual inspections were made during and after wave action on the structure. The wave height and category of overtopping (minor, moderate, or major) were noted for each wave condition. Minor overtopping was defined for the present study as occasional or not overtopping. Moderate overtopping was defined as regular overtopping with occasional green water. Conditions that produced frequent overtopping and green water were classified as major.

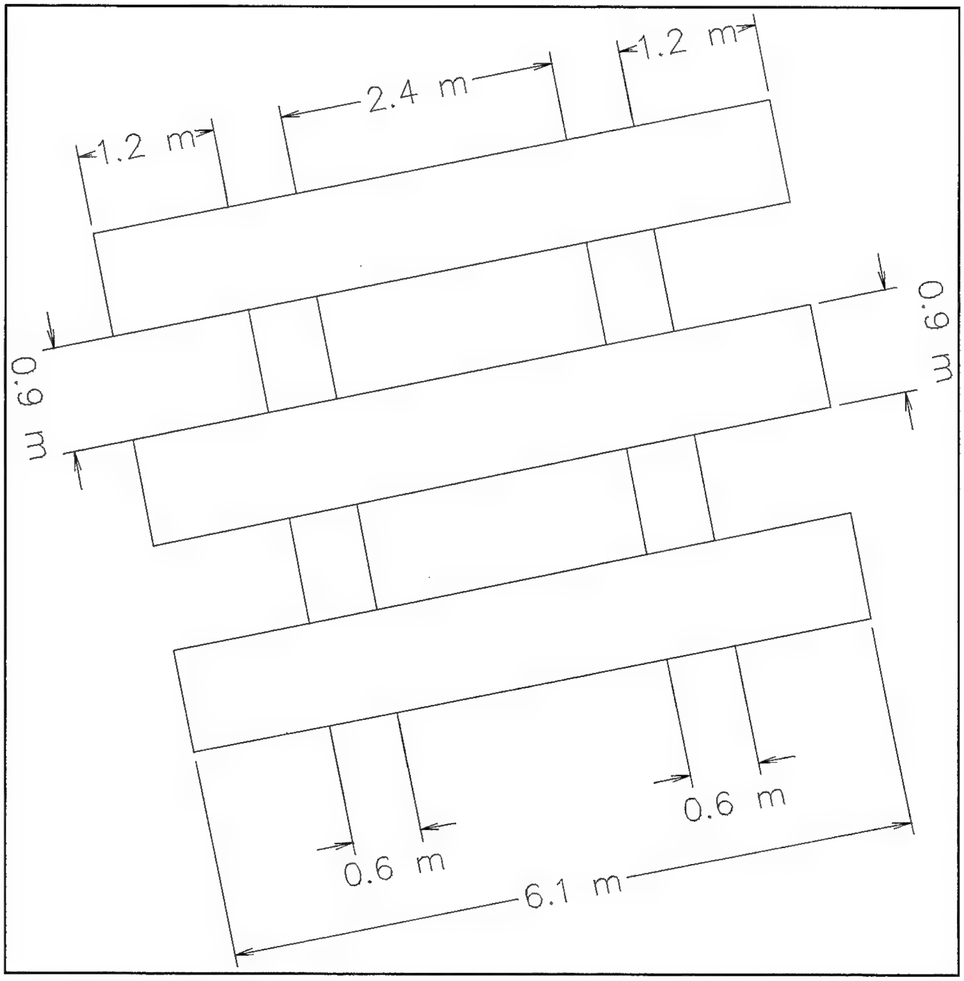

Figure 5. Dimensions of rib cap

3 Results

Five stability plans were studied. To gain an understanding on how the entire breakwater would respond to 2-D waves with a 1V:10H nearshore slope, deepwater wave conditions and the breakwater configuration of Plan 1H of the previous 3-D study (Smith 1998) were reproduced in Plan 1 of the present study. Subsequent plans were studied to determine the optimal breakwater design.

Plan 1

The breakwater cross section, developed from the previous 3-D model study, was constructed to a crest elevation of +6.1 m (+20 ft) mllw with a rib cap (Figures 6 through 9). Core-Locs having prototype weights of 17.8 and 32.2 tonnes (19.6 and 35.5 tons) were placed to -10.7 m and -8.2 m (-35 ft and -27 ft) mllw toe elevations on the sea side and lee side, respectively. Results in the 2-D experiment were similar to the 3-D study for waves up to the design wave (9.8 sec, 6.9 m (22.6 ft) at the structure), with one unit displaced. However, for waves greater than the design condition, incident waves shoaled on the 1V:10H slope and the higher waves in the series broke on the structure, which was not observed in the 3-D study. In the 3-D study, waves did not transform as they approached the structure and much of the energy was transferred onto the leeside units as waves surged over the structure. Twenty-three seaside units and one lee-side unit were displaced for 12-sec 8.5-m (28.0-ft) waves during experiments of Plan 1 (Figures 10 through 12).

The proposed breakwater has 98.4 linear m (323 linear ft) of breakwater trunk plus a head section. The 2-D model was 0.9 m (3 ft) wide at a 1:54.3 scale, which represents 49.6 m (163 ft) of the prototype breakwater trunk, or 50.4 percent of the prototype breakwater. If it is assumed that damage to the prototype trunk is proportional to the 2-D results with the nearshore slope included, it would be expected that 46 units would be displaced from the sea side and two units displaced from the lee side for the conditions studied. For identical offshore wave conditions, Plan 1H from the 3-D model study had 5 seaside units and 1 leeside unit displaced off the structure, which indicates that the influence of the nearshore slope is important.

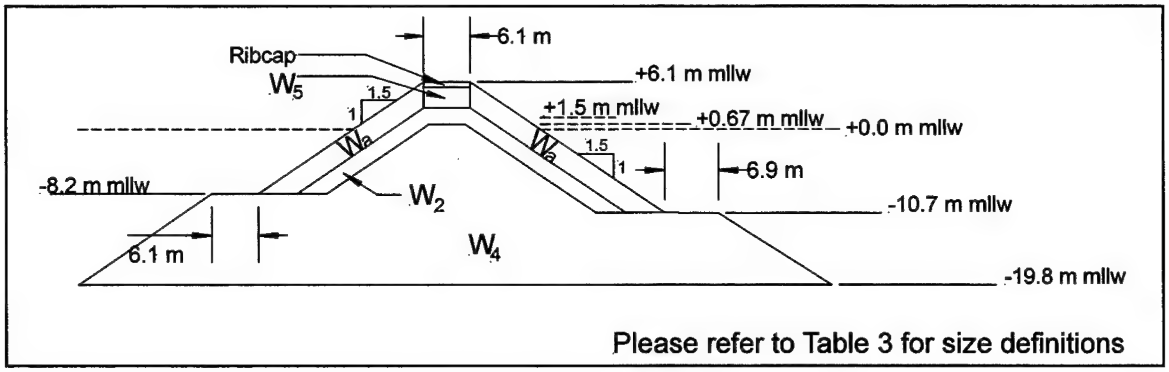

Figure 6. Cross section of Plans 1 and 2

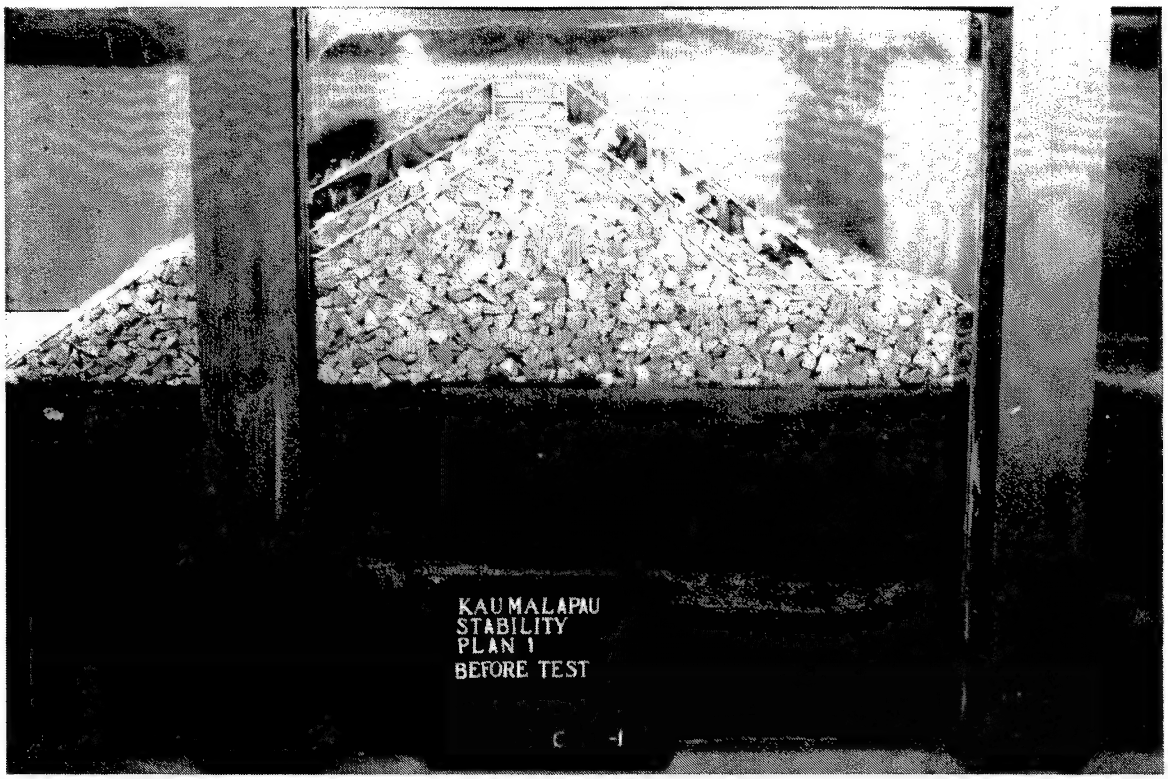

Figure 7. Side view of Plan 1, before experiment

Plan 2

The results of Plan 1 indicated that 17.8-tonne (19.6-ton) Core-Locs placed on the sea side are not sufficient to provide the desired breakwater stability. Except for Core-Loc layer thickness, Plan 2 had the same geometry as Plan 1 (Figure 6), but the structure was protected by 32.2-tonne (35.5-ton) Core-Locs and subjected to modified design wave conditions. Overtopping was minor for waves less than 4.6 m (15 ft), and moderate for 4.6-m (15-ft) waves. Occasional green water overtopped the breakwater for 12-sec, 4.6-m (15-ft) waves, and

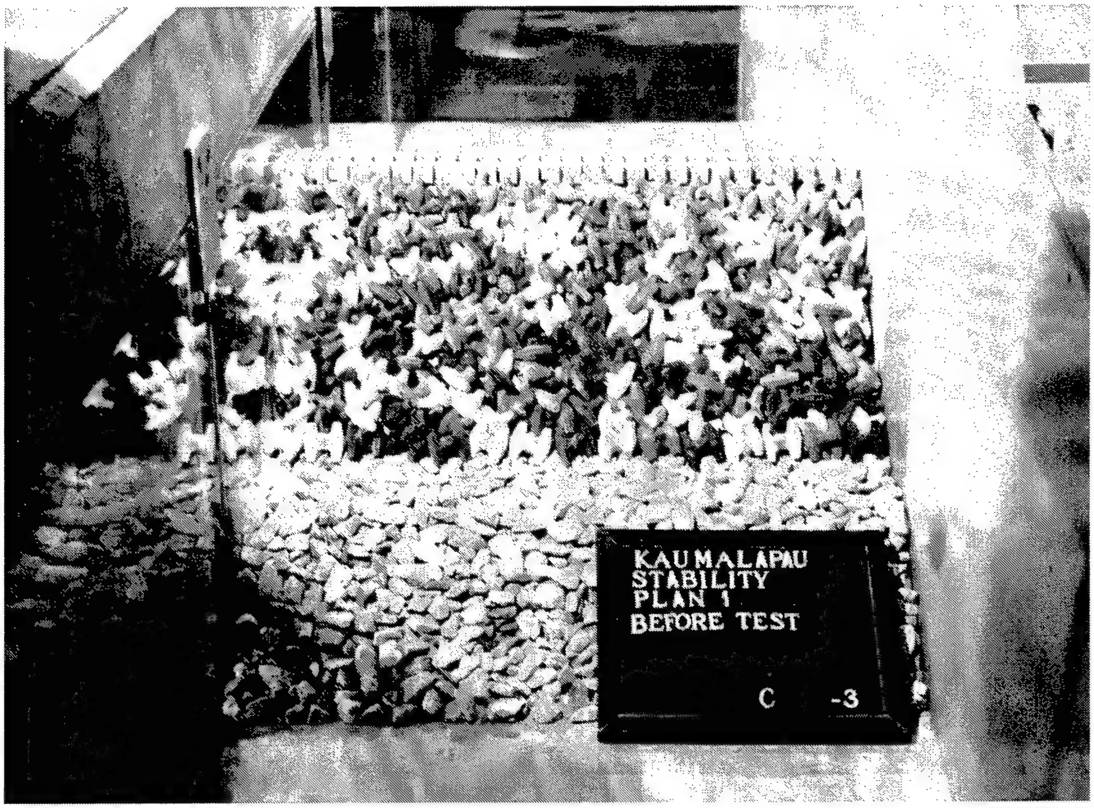

Figure 8. Sea-side view of Plan 1, before experiment

Figure 9. Leeside view of Plan 1, before experiment

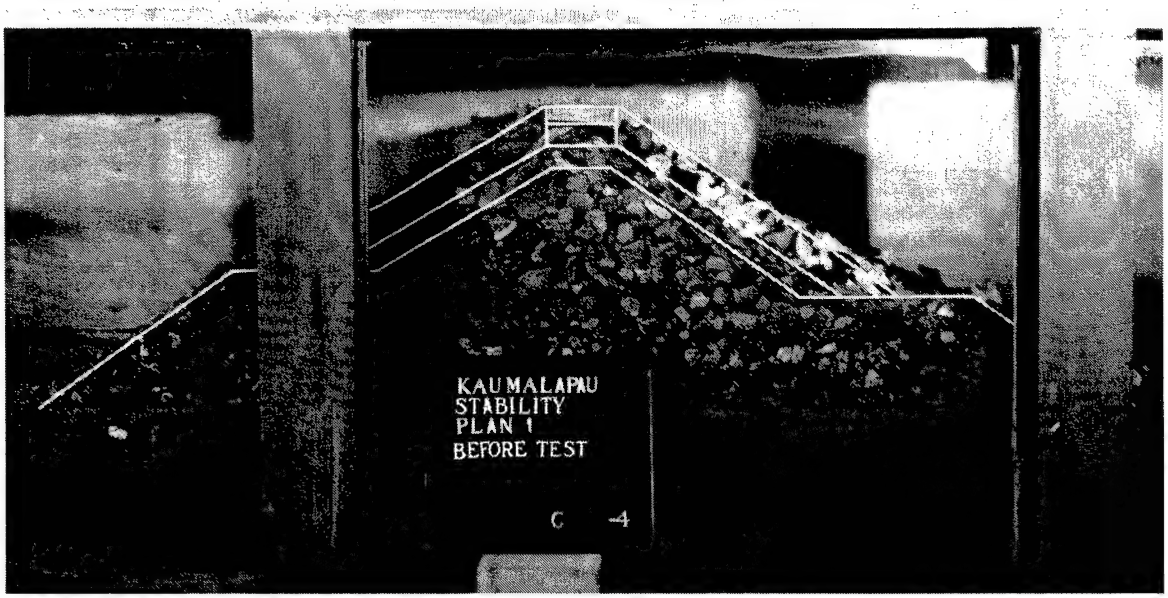

Figure 10. Side view of Plan 1, after experiment

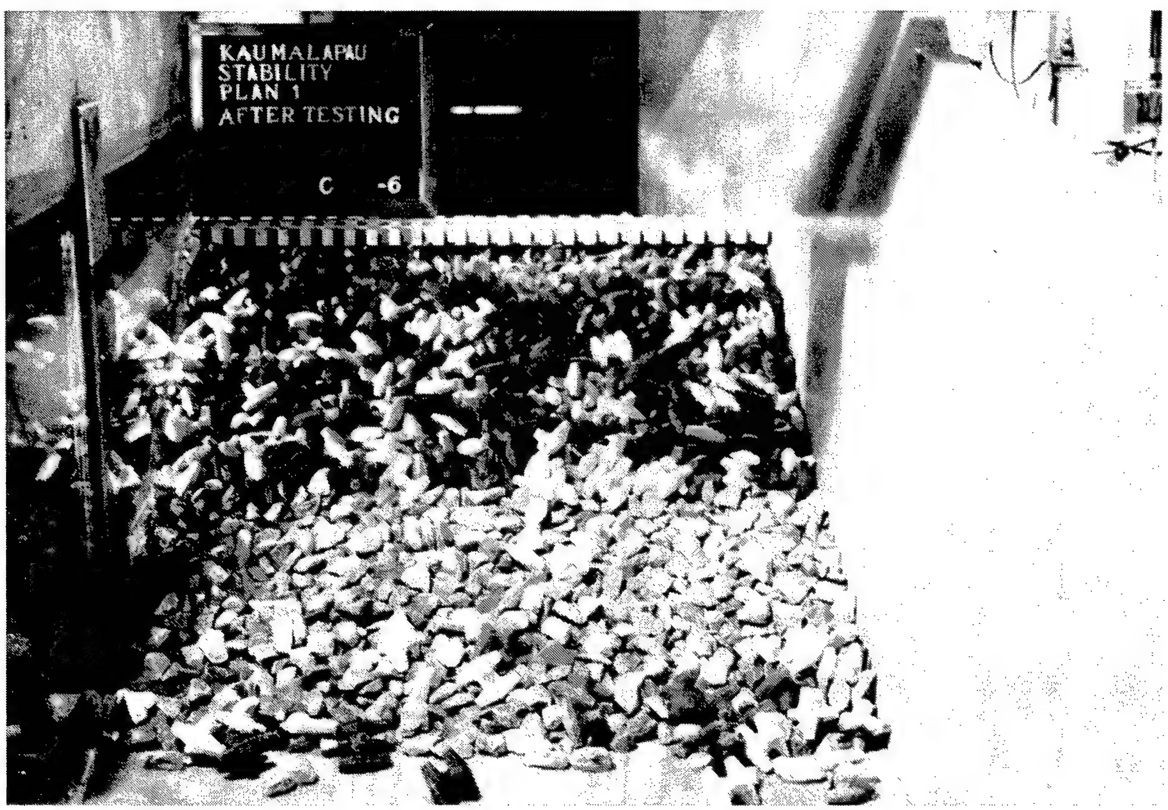

Figure 11. Sea-side view of Plan 1, after experiment

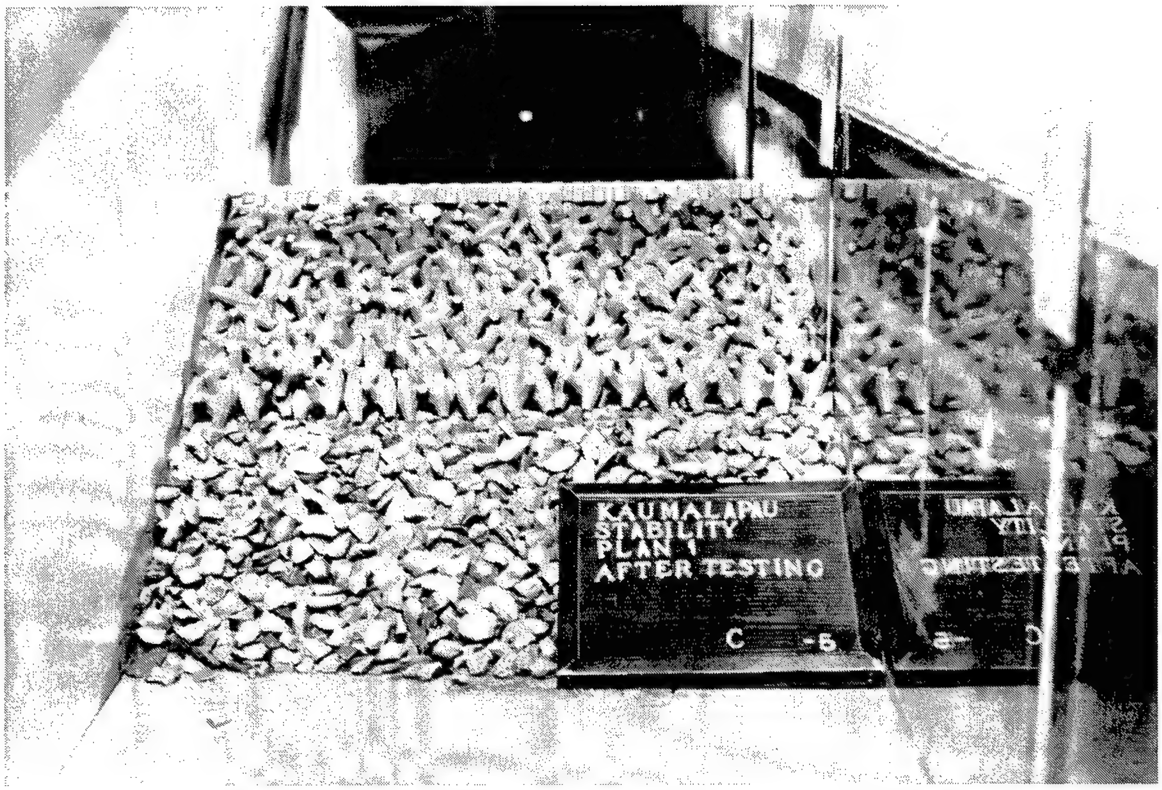

Figure 12. Leeside view of Plan 1, after experiment

green water was more frequent with major overtopping for 6.1-m (20-ft) waves and higher. The sea-side toe stones began to rock in place for 12-sec, 6.1-m (20-ft) waves, but the structure remained undamaged for waves up to 12 sec, 7.6 m (25 ft). Toe stones were displaced and the Core-Loc toe became unstable for 12-sec, 9.1-m (30-ft) waves. Three units were displaced off the sea side during the 12-sec, 9.1-m (30-ft) wave condition. One additional sea-side unit was displaced with 16-sec, 9.1-m (30-ft) waves, but damage was minor. It was observed that the wave rundown extended to the sea-side toe units with higher waves. Five additional units were displaced off the sea side with 12-sec, 10.7-m (35-ft) waves, but the structure remained intact. The structure was destroyed with 16-sec, 12.2-m (40-ft) waves. Sea-side units were displaced, exposing the underlayer, which also was displaced, causing a breach to occur to the lee side. (Photographs unavailable for Plan 2)

Plan 3

From observations during Plan 2, instability at the toe caused the armor layer to slip and loosen the interlocking between Core-Locs. To determine if 32.2-tonne (35.5-ton) Core-Locs were stable, instability at the toe was minimized for Plan 3 by extending the sea-side Core-Loc toe to -19.8 m (-65 ft) mllw, fronted by a stone toe berm composed of 2.7-to 5.1-tonne (3.0- to 5.6-ton) stone (Figure 13). The toe of the lee side was stable on Plan 2, therefore the elevation

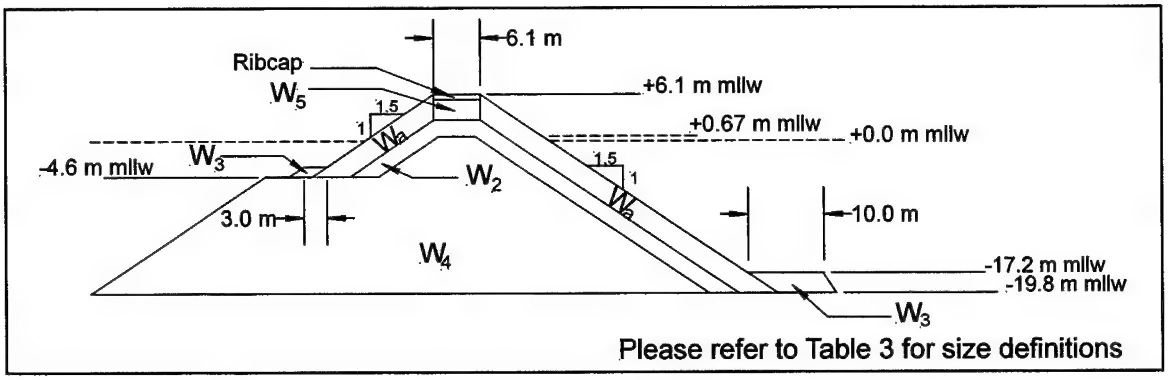

Figure 13. Cross section of Plan 3

of the leeside toe was raised to -4.6 m (-15 ft) mllw and included a 3.0-m-(10-ft-) wide toe buttress of 2.7 to 5.1-tonne (3.0- to 5.6-ton) stone. The crest elevation remained at +6.1 m (+20 ft) mllw. Overtopping was similar to Plan 2; minor for waves less than 4.6 m (15 ft), moderate for 4.6-m (15-ft) waves, and major for 6.1-m (20-ft) waves and higher. Individual toe stones were displaced with 16-sec, 9.1-m (30-ft) waves, and the sea-side toe berm became rounded, but was stable. In general, little displacement of toe stones occurred on the sea side. However, overtopping waves began displacing leeside toe stones with 12-sec 7.6-m (25-ft) waves and continued to damage the leeside toe berm with higher waves. The structure remained stable through 12-sec 10.7-m (35-ft) waves, a total of two units were displaced on the sea side. However, many sea-side units were observed to rock in place for higher waves of individual wave series. The leeside units failed with 16-sec, 12.2-m (40-ft) waves due to toe failure caused by overtopping waves. The sea side appeared to be stable until a breach occurred from the lee side (Figures 14 through 16).

Plan 4

Experiments were conducted on Plan 4, which consisted of 32.2-tonne (35.5-ton) Core-Locs. The seaside toe extended to -13.7 m (-45 ft) mllw and included a toe berm constructed of 2.7 to 5.1-tonne (3.0 to 5.6 ton-) stone, 3.0 m (10 ft) wide. The leeside toe extended to -6.1 m (-20 ft) mllw and also included a 2.7 to 5.1 tonne (3.0 to 5.6 ton-) stone toe berm, 9.1 m (30 ft) wide. Additionally, the crest elevation was lowered from +6.1 m to +4.6 m (+20 ft to +15 ft) mllw (Figures 17 through 20). Minor overtopping occurred with 3.0-m- (10-ft-) waves with occasional water splashing onto the rib cap. Overtopping was moderate with occasional overtopping green water with 9.8-sec, 4.6-m (15-ft) waves. Major overtopping occurred with 6.1-m- (20-ft-) waves and higher. The structure was stable with minor damage on the sea side (one unit displaced, several rocking in place) with waves up to 12 sec, 12.2 m (40 ft). Toe stones began to rock in place on the sea-side berm with 12-sec, 4.6-m (15-ft) waves, and 12-sec, 7.6-m (25-ft) waves began to reshape the sea-side toe berm. Several toe stones were displaced with 12- and 16-sec, 10.7-m (35-ft) waves, but only one armor unit was displaced during 16-sec, 10.7-m (35-ft) waves, and the

Figure 14. Side view of Plan 3, after experiment

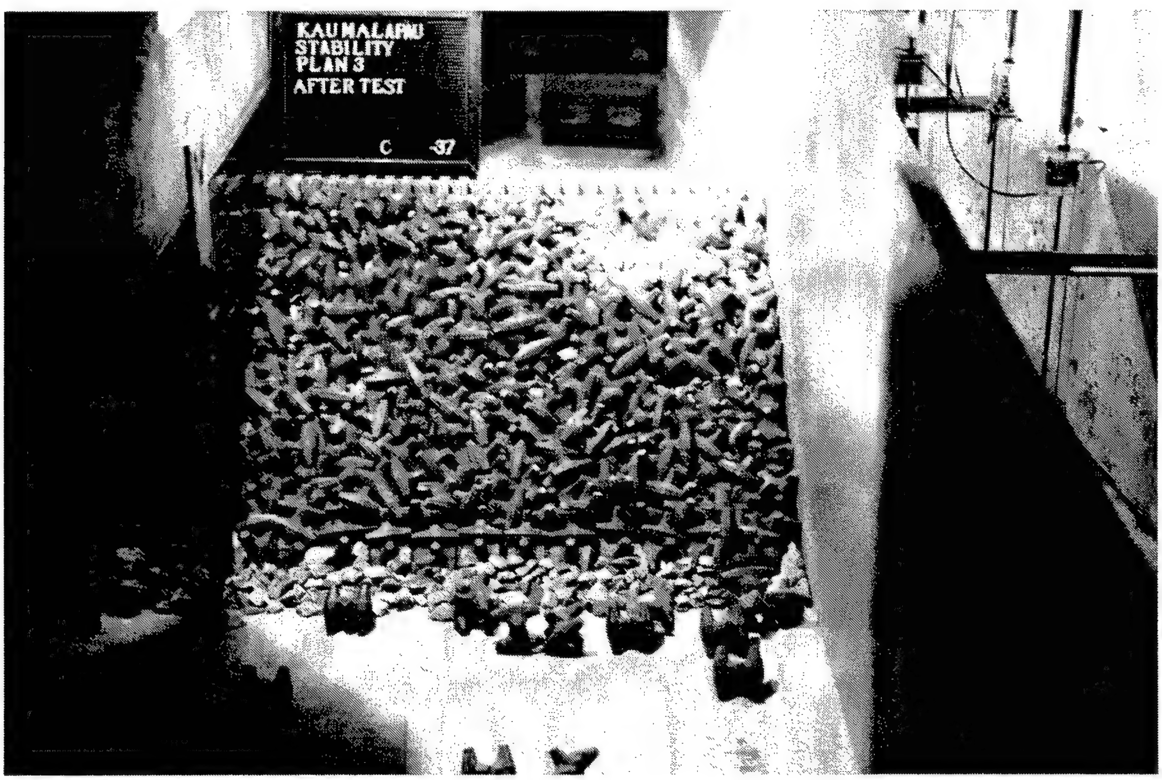

Figure 15. Sea-side view of Plan 3, after experiment

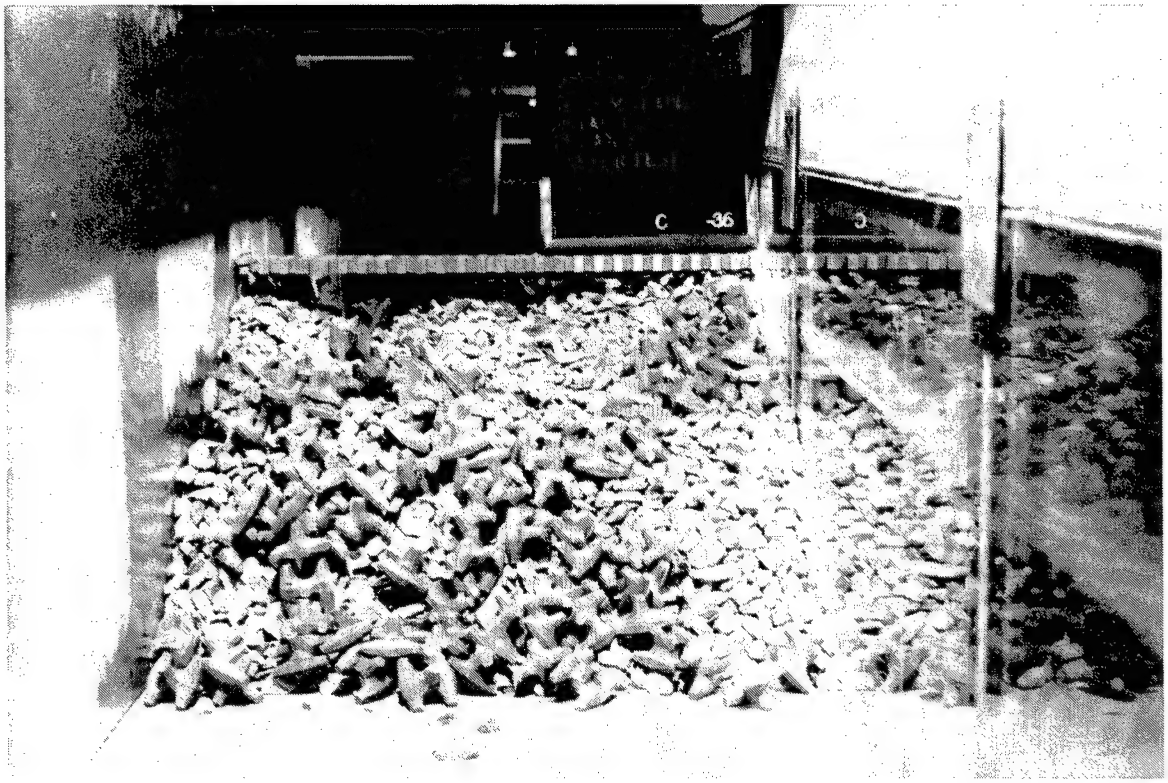

Figure 16. Leeside view of Plan 3, after experiment

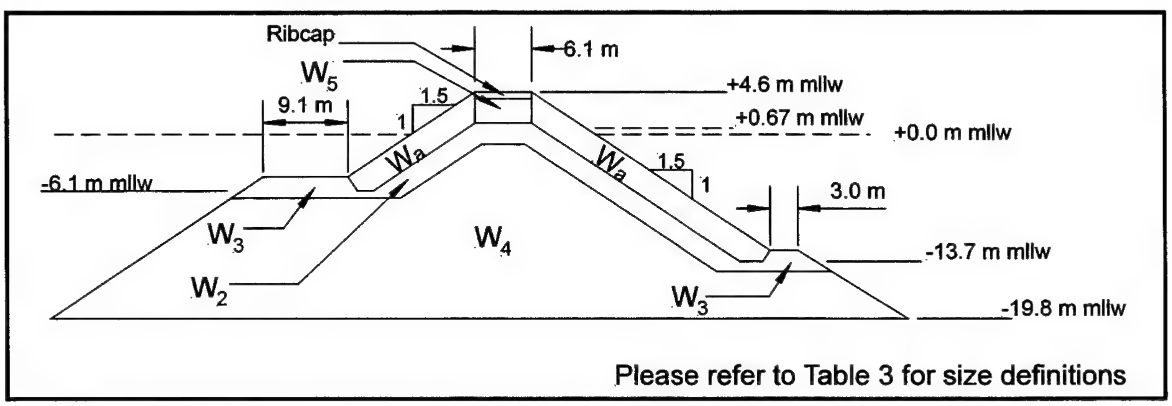

Figure 17. Cross section of Plan 4

breakwater remained stable. Overtopping 16-sec, 10.7-m (30-ft) waves displaced leeside toe stones. The structure was moderately damaged with 16-sec, 12.2-m (40-ft) waves: 10 sea-side units and 2 leeside units were displaced (Figures 21 through 23). A portion of the damage appeared to result from looseness of the armor layer due to loss of toe stones.

Figure 18. Side view of Plan 4, before experiment

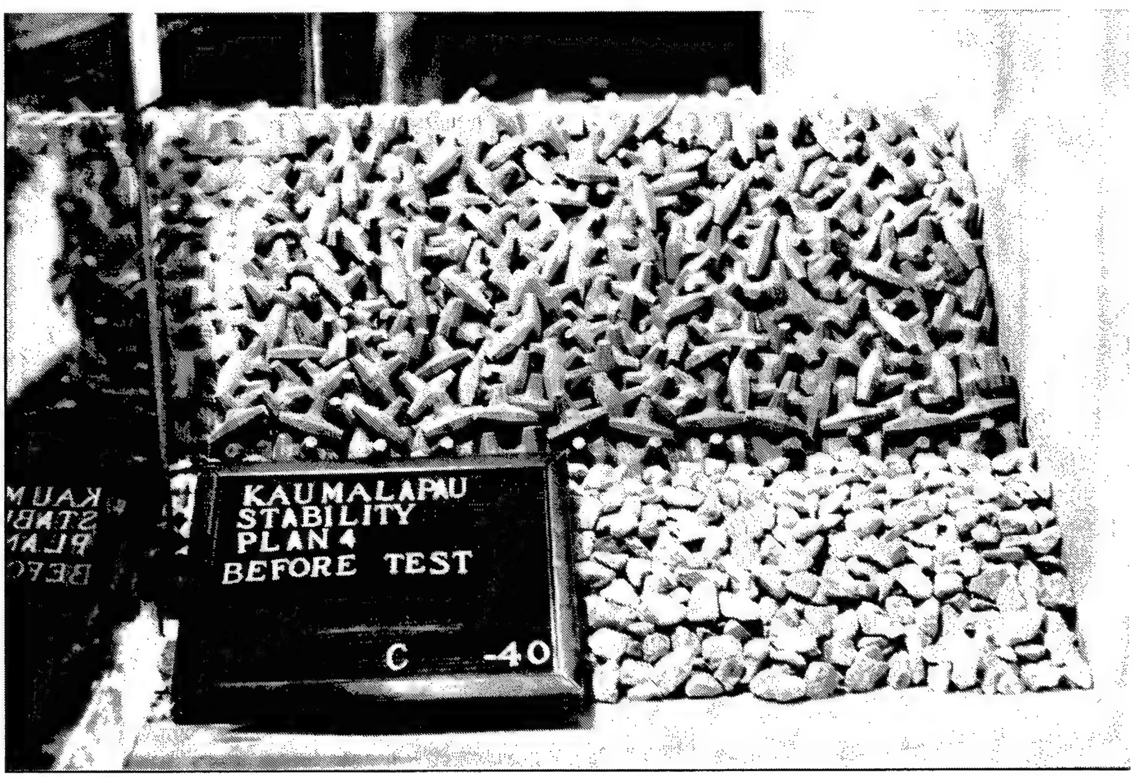

Figure 19. Sea-side view of Plan 4, before experiment

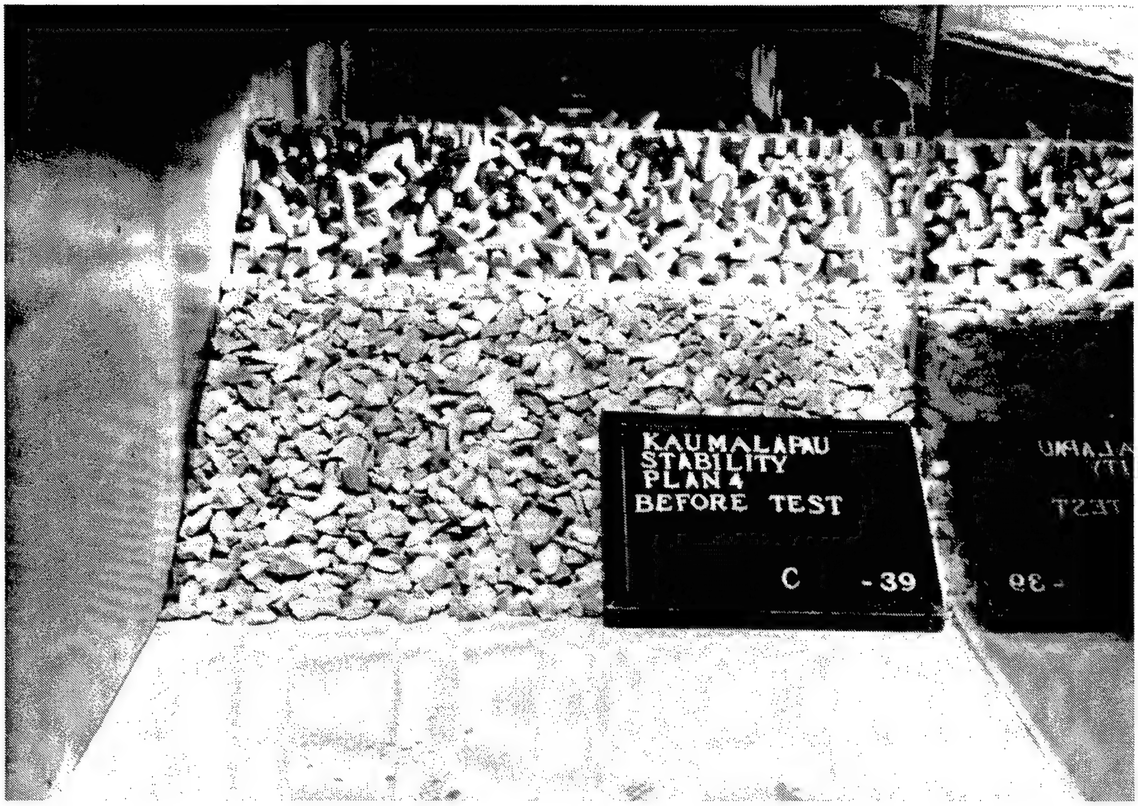

Figure 20. Leeside view of Plan 4, before experiment

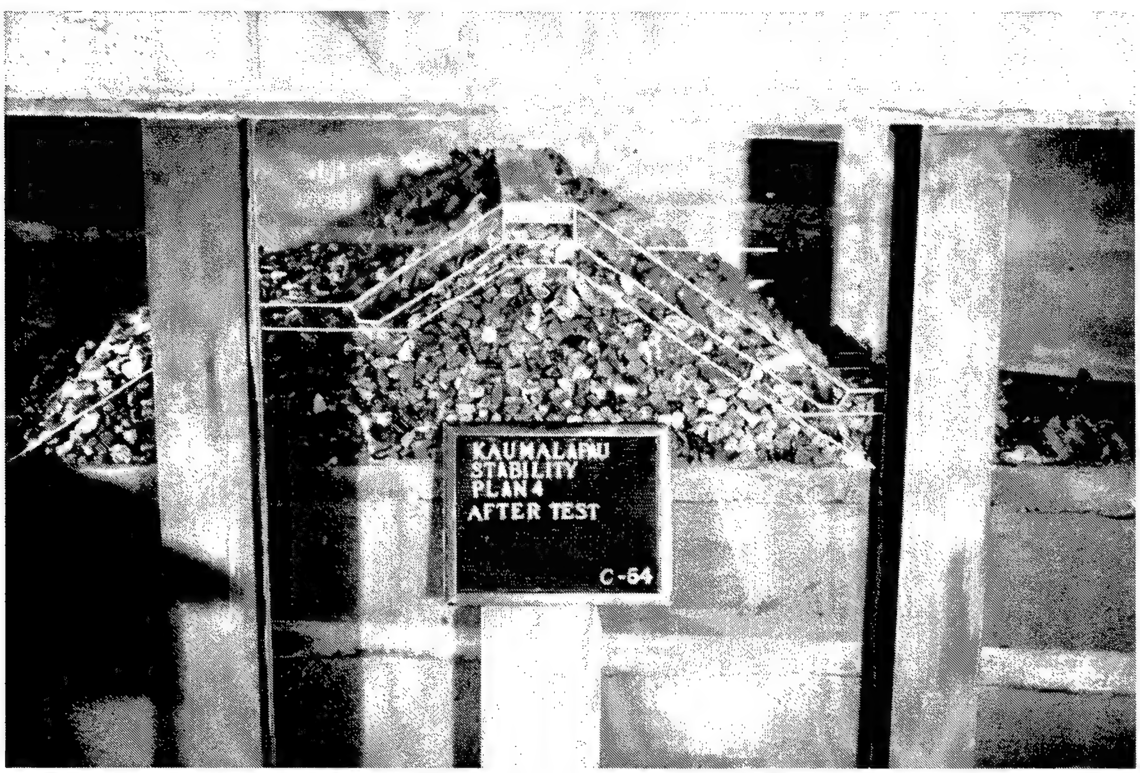

Figure 21. Side view of Plan 4, after experiment

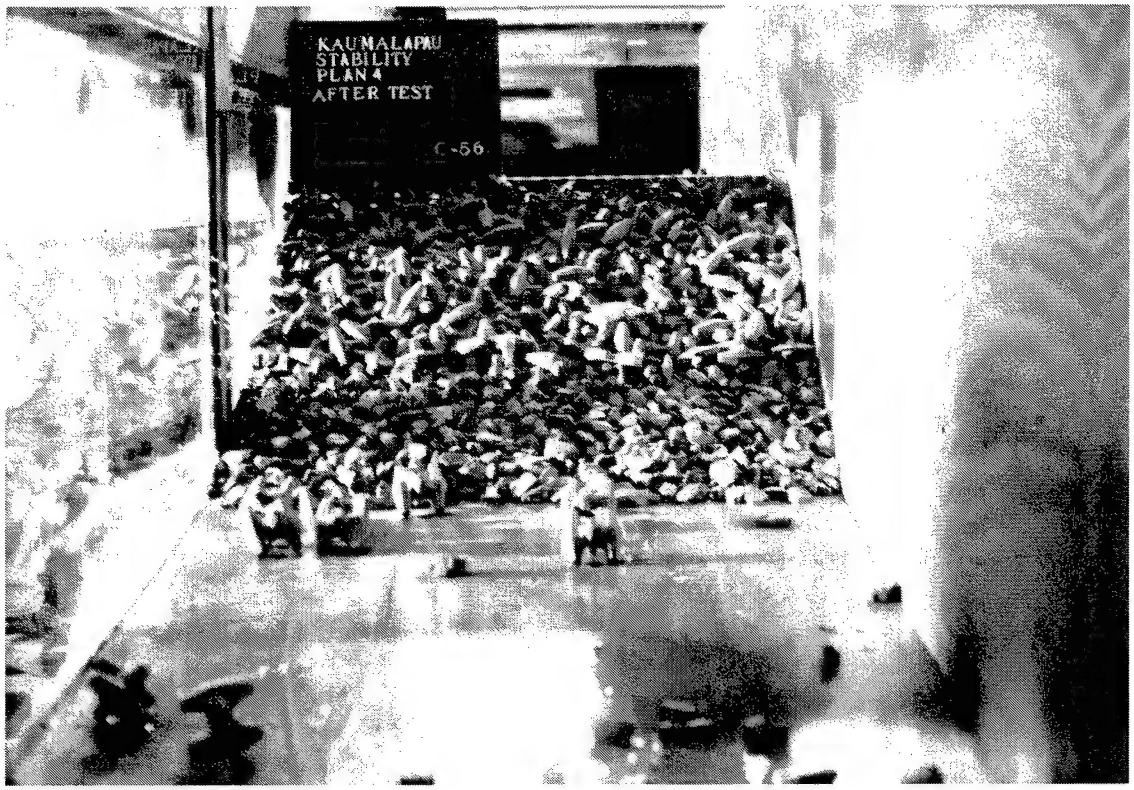

Figure 22. Sea-side view of Plan 4, after experiment

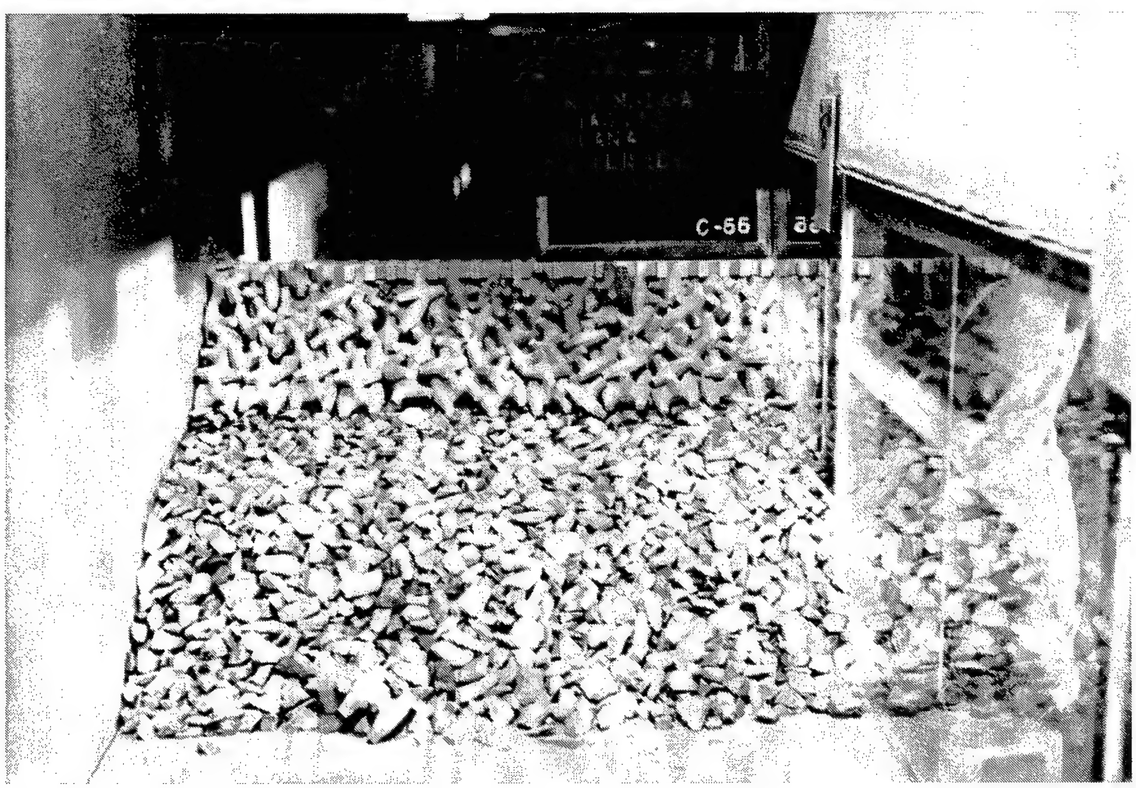

Figure 23. Leeside view of Plan 4, after experiment

Plan 5

Plan 5 was constructed using 32.2-tonne (35.5-ton) Core-Locs in the same configuration as Plan 4 except the crest elevation was lowered from +4.6 m (+15 ft) mllw to +3.7 m (+12 ft) mllw (Figure 24). Plan 5 was subjected to modified design wave conditions. Moderate overtopping occurred for 12-sec, 3.0-m (10-ft) waves. Overtopping was moderate to major for 12-sec, 4.6-m (15-ft) waves and major for 6.1-m (20-ft) waves and higher. Sea-side toe stones were observed to rock in place during 12-sec, 4.6-m (15-ft) waves and stones began to be displaced for 12-sec, 6.1-m (20-ft) waves. A scour hole began to form on the leeside toe berm for 16-sec, 7.6-m (25-ft) waves overtopping the structure and continued to be displaced for subsequent wave series of higher magnitude. Damage was minor after 12-sec, 12.2-m (40-ft) waves: three sea-side units and two leeside units displaced. Plan 5 had moderate to major damage after 16-sec, 12.2-m (40-ft) waves with a total of 18 sea-side and 4 leeside units displaced (Figures 25 through 27).

It was observed during experiments of Plans 4 and 5 that several units rocked in place for wave heights greater than or equal to 9.1 m (30 ft), which could possibly equate to breakage in the prototype.

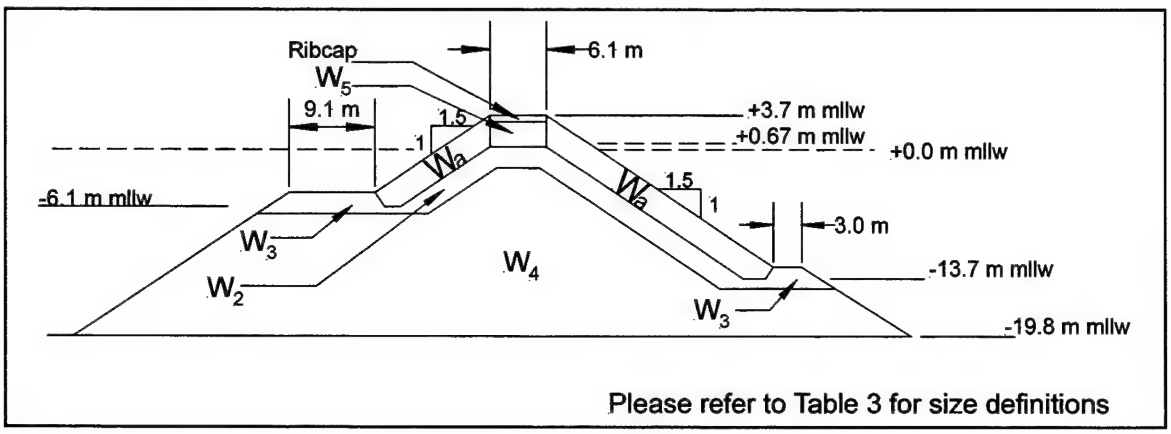

Figure 24. Cross section of Plan 5

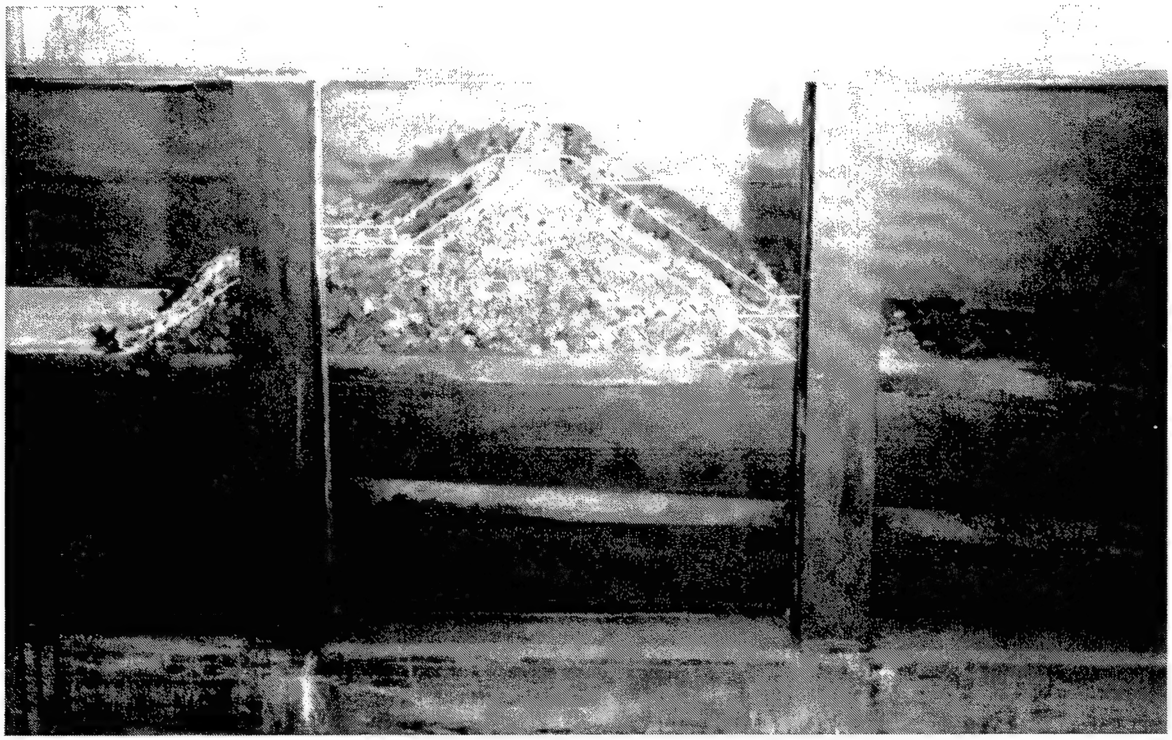

Figure 25. Side view of Plan 5, after experiment

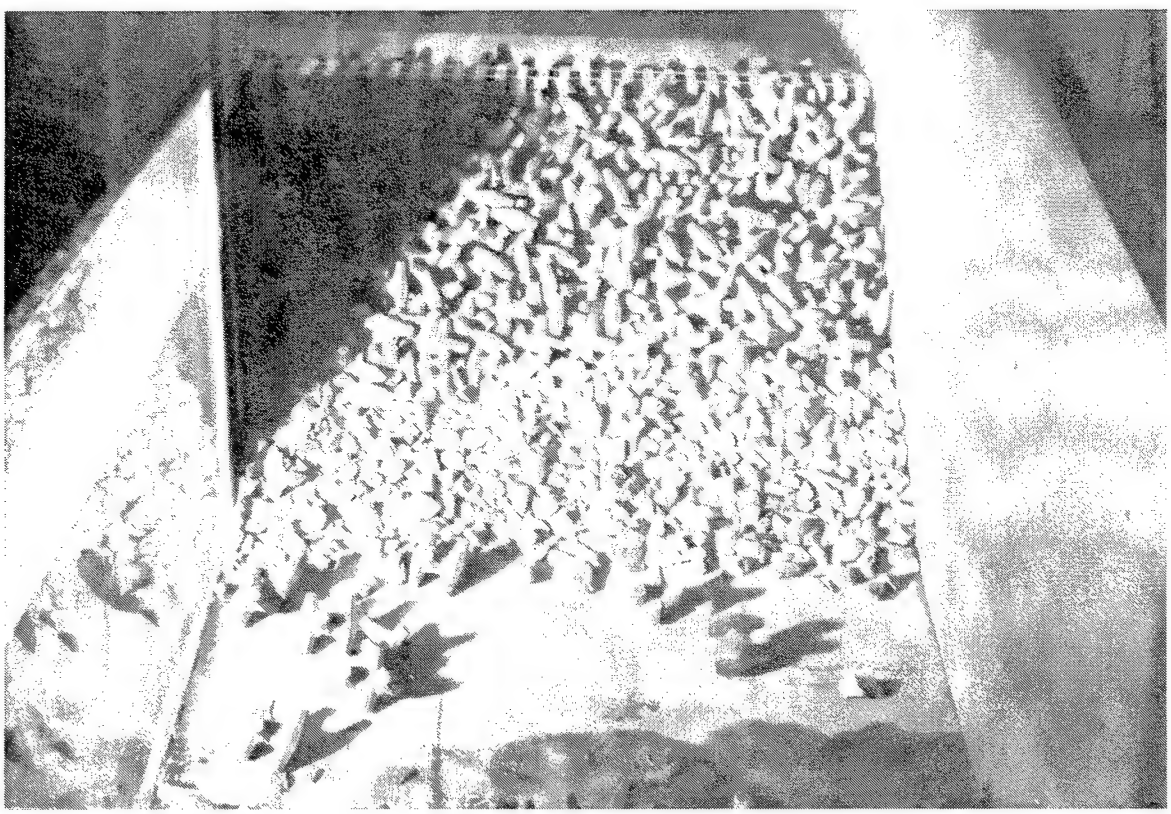

Figure 26. Sea-side view of Plan 5, after experiment

Figure 27. Leeside view of Plan 5, after experiment

4 Conclusions and Discussion

Conclusions

From the 2-D experiment results, it was determined that:

a. The Core-Loc size, 17.8 tonnes (19.6 tons), used on the sea side in the 3-D study was not stable for 12-sec, 8.5-m (28-ft) waves.

b. The heavier, 32.2-tonne (35.5-ton), Core-Loc appeared to be stable with no damage for waves up to 10.7 m (35 ft) if the depth of the armor toe was placed in sufficiently deep water (-19.8 m (-65 ft) mllw on the sea side, and -6.1 m (-20 ft) mllw on the lee side). On the sea side, toe stones were observed to rock in place and displace for incident waves in which the wave drawdown approached the depth of the toe. Displacement of toe stones led to subsidence of the armor layer and loss of interlocking between Core-Locs, which led to substantial damage to the breakwater. On the lee side, energy from overtopping waves displaced toe stones placed high in the water column, which led to leeside toe failure.

c. If 32.2-tonne (35.5-ton) sea-side units were placed at a -13.7 m (-45 ft) mllw toe elevation, the sea side of the breakwater suffered minor damage with 12-sec, 12.2-m (40-ft) waves, and moderate to major damage with 16-sec, 12.2-m (40-ft) waves. No displacement was observed with 10.7-m (35-ft) waves. Minor damage occurred on the lee side units with 10.7-m (35-ft) waves if the toe was placed at -6.1 m (-20 ft) mllw.

d. The 32.2-tonne (35.5-ton) Core-Locs showed significant rocking in place during waves 7.6 m (25 ft) and higher. This could possibly result in armor unit damage.

e. Observations of wave overtopping during the study indicated that if the crest elevation was +4.6 m (+15 ft) mllw or + 6.1 m (+20 ft) mllw, overtopping was minor for incident waves less than 4.6 m (15 ft), moderate for 4.6-m (15-ft) waves, and major for 6.1-m (20-ft) waves and higher for swl of 0.67 m (+2.2) ft mllw. The difference in crest elevation between +4.6 m (+15 ft) mllw and +6.1 m (+20 ft) mllw did not significantly change the amount of water overtopping the structure. For

experiments with a crest elevation of +3.7 m (+12 ft) mllw, moderate overtopping occurred for 3.0-m (10-ft) waves, and major overtopping occurred for 4.6-m (15-ft) waves and higher for a swl of +0.67 m (+2.2 ft) mllw.

Results from the 2-D study indicated the most stable plan was Plan 4, which consisted of 32.2-tonne (35.5-ton) Core-Locs placed from a crest elevation of +4.6 m (+15 ft) mllw to a sea-side toe elevation of -13.7 m (-45 ft) mllw and a leeside toe elevation of -6.1 m (-20 ft) mllw. The armor toe was protected by a toe berm constructed of 2.7- to 5.1-tonne (3.0- to 5.6-ton) stone placed 3.0 m (10 ft) wide on the sea side, and 9.1 m (30 ft) wide on the lee side.

Discussion

It is difficult to extrapolate the damage that would be expected to the head section from the 2-D results. Both the head and trunk of Plan 1H from the 3-D study (Smith 1998) were considered not damaged (less than 2 percent displacement by count). However, Turk and Melby (1997) recommend stability coefficients, K_d's, for design should be 16 for the trunk section and 13 for the head section. If it is assumed that the ratio of the head to trunk stability coefficients, (13/16th or 81.25 percent) applies for all conditions, an estimate of the head stability can be inferred. The stability coefficient is calculated using the Hudson equation:

$$K_d = \frac{\gamma_a H_d^3}{W_a (S_a - 1)^3 \cot\theta} \tag{3}$$

in which K_d is the stability coefficient, H_d is the highest zero-moment wave height at the structure that causes no damage, i.e., wave height at which damage is less than or equal to 2 percent, and θ is the structure slope measured from horizontal in degrees. From Plan 4 experiments H_d was 11.1 m (36.3 ft), which yields a K_d-value of 32.6. By assuming the head section stability coefficient is 81.25 percent of the trunk, $K_d = 26.5$ at the head section.

If 32.2-tonne (35.5-ton) units are placed on the head and the estimated K_d-value of 26.5 is used in Equation 3, $H_d = 10.3$ m (33.9 ft) at the head. To maintain the same stability for both the trunk and head sections with 11.1-m (36.3-ft) waves, it would be required to increase Core-Loc size, reduce the slope of the head section, or a combination of the two. For example, an increase in Core-Loc weight to 39.5 tonnes (43.6 tons) or a milder slope of 1V:1.8H at the head section would result in $H_d = 11.1$ m (36.3 ft) using Equation 3.

It should be emphasized that this inference is based on 2-D results of the trunk with normally incident waves. Three-dimensional effects such as bathymetrical changes or direction of wave approach were not modeled and cannot be predicted. Additionally, nothing can be inferred at the landward end of breakwater from the 2-D results.

Although Plan 4 was the recommended plan, it may be desired to alter the design to minimize construction costs. For selected wave and water levels, the 2-D study showed that the sea-side toe elevation should be constructed to at least -13.7 m (-45 ft) mllw and the leeside toe elevation should be constructed to -6.1 m (-20 ft) mllw to provide adequate stability for the design wave conditions. Although the breakwater lee side remained stable for a crest elevation of +3.7 m (+12 ft) mllw, overtopping increased for experiments conducted with the crest at this elevation. Therefore, it is recommended that the crest elevation be constructed at or near +4.6 m (+15 ft) mllw.

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Appendix A
Notation

a	Area scale
A	Area
H_d	Average zero-moment wave height at structure
H_{mo}	Zero-moment wave height
H_o	Deepwater wave height
H_s	Significant wave height
K_d	Stability coefficient
l	Length scale
m	Model quantity
N	Number of units in a given area
p	Protoype quantity
r	Subscript denoting model to prototype
S_a	Specific gravity of an individual armor unit relative to the water in which it is placed, $S_a = \gamma_a/\gamma_w$
t	Time scale
T_p	Peak wave period
v	Volume scale
V	Volume of armor unit

W_2 Weight of stone in first underlayer

W_3 Weight of toe stone

W_4 Weight of existing material (core)

W_5 Weight of armor cap stone

W_a Weight of an individual armor unit

θ Structure slope measured from horizontal in degrees

ϕ Packing density coefficient

γ_a Specific weight of an individual armor unit

γ_w Specific weight of water

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14. ABSTRACT

A two-dimensional breakwater study was conducted to examine the effects of design and above wave conditions on the stability of the proposed breakwater rehabilitation at Kaumalapau Harbor, Lanai, HI. Additionally, it was desired to determine what effect, if any, cost reducing construction modifications would have on breakwater stability. The modifications examined were combinations of reducing the crest elevation and raising the armor layer toe elevation. Results of the study were compared to results of a previous three-dimensional study of Kaumalapau Harbor (Smith 1998). Inferences were made to estimate the effects the modified wave and bathymetrical conditions would have on stability of the breakwater head.

Five stability plans were studied in a wave tank at a 1:54.3 (model to prototype) scale. The model consisted of a 1V:10H nearshore slope to simulate prototype bathymetry, which was not included in the three-dimensional study of Smith (1998). All plans were constructed using Core-Loc armor units. Plan 1, the recommended plan from the three-dimensional study, was constructed of 17.8-tonne Core-Locs on the sea side and 32.2-tonne on the lee side. Results from Plan 1 experiments indicated that the 17.8-tonne Core-Locs were not stable for 12-sec, 8.5-m waves. Plans 2 through 5 were studied to determine the optimal breakwater design using 32.2-tonne Core-Locs on both the sea side and the lee side. The heavier armor unit appeared to be stable with no damage for waves up to 10.7 m if the depth of

(Continued)

15. SUBJECT TERMS

Breakwaters	Core-Loc	Hydraulic models	Overtopping
Breakwater stability	Harbors, Hawaii	Kaumalapau, Hawaii	Physical models

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the armor toe was placed in sufficiently deep water (-19.8 m mean lower low water (mllw) on the sea side and –6.1 m mllw on the lee side). However, the model armor units were observed to rock in place when exposed to waves 7.6 m and higher, indicating that it is possible that for these conditions, breakage of the armor units could occur in the prototype. It was observed in the model that wave overtopping with a crest elevation of +4.6 m mllw was minor for waves less than 4.6 m, moderate for 4.6 m-waves, and major for 6.1-m waves and higher for a still-water level of +0.67 m mllw.